PRELIMINARY DRAINAGE REPORT

FOR

PDS2013 – TM - 5577
ER LOG NO. PDS 2013 – ER – 13 – 02 - 003
1650 WINTERHAVEN ROAD, FALLBROOK, CA 92028
APN 106 280 22

PREPARED FOR

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October 28, 2014

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Robert Sukup

The proposed project is a 21 lot subdivision in Fallbrook with 1.0 acre minimum size lots. The property is 26.48 acres. There is a very small amount of water that enters the property from the north from 2 existing 1 acre lots. There is also a small area to the west that enters the property. Further to the west there is the potential for about 4.2 acres to flow to Winterhaven Road that would then flow via a spillway to the existing 30" CMP storm drain that flows under Winterhaven and exits on the southern side of the street.

The purpose of this study was to identify all the drainage courses, determine the volume of discharges, calculate the size of new pipes, and determine the adequacy of existing pipes.

Based on my calculations, the existing 24" CMP under Winterhaven will have to be replaced by a 27" RCP pipe, at a minimum. The existing 30" CMP is adequate. It needs to be replaced due to its limited life expectancy.

The two onsite storm drain pipes are designed to be 24" RCP's and are adequate.

I have attached 3 sheets of Exhibits that indicate all the different drainage areas and flows for the project.

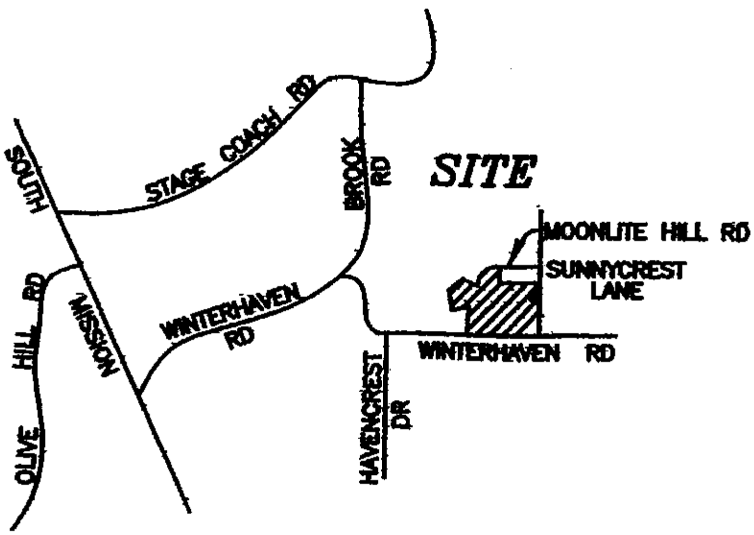
This project is a Priority Project based on the Stormwater Checklist. The proposed project will increase the projected flow from the site and was identified within the calculations.

To mitigate this increase in flow, an intense SWMM Report for both Water Quality and Hydromodification (HMP) BMP's was prepared as a separate document. I have attached the first 4 pages of the Hydromodification Analysis and the cross sections for the Bioretention Facilities. These 6 pages are attached at the end of the report and are numbered 1/6 to 6/6. Bioretention Basin 1 is for Drainage Area 1A and Bioretention Basin 2 is for Drainage Area 2 which outlets at the 2D location in my hydrology exhibits.

Basin 1 has a low flow control plate with a 2" orifice. Once the capacity of the orifice and basin storage area is exceeded, the drainage water enters into a 3'x3' grated inlet that runs the excess discharge into the outlet structure and then through the high flow outlet pipe (24" RCP).

Basin 2 has 2 low flow control plates with a 3.75" orifice on each. The basin then has a middle orifice outlet which is 9" above the soil layer. This orifice consists of 10- 4" holes which enter into the sides of the outlet structure. Once the capacity of the low orifice, the medium orifice, and basin storage area is exceeded, the drainage water enters into a 3'x3' grated inlet that runs the excess discharge into the outlet structure and then through the high flow outlet pipe (27" RCP).

All of the above is to be implemented in the final design of the project. It is my professional opinion that no downstream flooding or siltation should occur because of this project.



VICINITY MAP

NO SCALE


R. 4 W.
R. 3 W.
1 700 000 FEET
San
13
12
24
19
20
21
25
29
36
MACDONALD
PECHE
STAGE
WINTER
MISSION
CANYON
GREEN
SITE
VIA
ENCINOS
RD
FaD2
FvD
PeC
StG
BlC
FaC
FaC2
FaE2
VsE
VsC
VaB
PeD2
PfC
ClE2
RaD2
RaC
VsG
CmrG
ClG2
PeC2
FvE
FeE
RaB
VsD
BlD2
FaB

Table 3-1
RUNOFF COEFFICIENTS FOR URBAN AREAS

Land Use		Runoff Coefficient "C"				
			Soil Type			
NRCS Elements	County Elements	% IMPER.	A	B	C	D
Undisturbed Natural Terrain (Natural)	Permanent Open Space	0*	0.20	0.25	0.30	0.35
Low Density Residential (LDR)	Residential, 1.0 DU/A or less	10 14%	0.27	0.32 ←.34	0.36	0.41
Low Density Residential (LDR)	Residential, 2.0 DU/A or less	20	0.34	0.38	0.42	0.46
Low Density Residential (LDR)	Residential, 2.9 DU/A or less	25	0.38	0.41	0.45	0.49
Medium Density Residential (MDR)	Residential, 4.3 DU/A or less	30	0.41	0.45	0.48	0.52
Medium Density Residential (MDR)	Residential, 7.3 DU/A or less	40	0.48	0.51	0.54	0.57
Medium Density Residential (MDR)	Residential, 10.9 DU/A or less	45	0.52	0.54	0.57	0.60
Medium Density Residential (MDR)	Residential, 14.5 DU/A or less	50	0.55	0.58	0.60	0.63
High Density Residential (HDR)	Residential, 24.0 DU/A or less	65	0.66	0.67	0.69	0.71
High Density Residential (HDR)	Residential, 43.0 DU/A or less	80	0.76	0.77	0.78	0.79
Commercial/Industrial (N. Com)	Neighborhood Commercial	80	0.76	0.77	0.78	0.79
Commercial/Industrial (G. Com)	General Commercial	85	0.80	0.80	0.81	0.82
Commercial/Industrial (O.P. Com)	Office Professional/Commercial	90	0.83	0.84	0.84	0.85
Commercial/Industrial (Limited I.)	Limited Industrial	90	0.83	0.84	0.84	0.85
Commercial/Industrial (General I.)	General Industrial	95	0.87	0.87	0.87	0.87

*The values associated with 0% impervious may be used for direct calculation of the runoff coefficient as described in Section 3.1.2 (representing the pervious runoff coefficient, Cp, for the soil type), or for areas that will remain undisturbed in perpetuity. Justification must be given that the area will remain natural forever (e.g., the area is located in Cleveland National Forest).
DU/A = dwelling units per acre
NRCS = National Resources Conservation Service

BERK HYDROLOGY CALCS

APN- 106-280-22

SOIL TYPE = "B" SEE ATTACHED PORTION OF BONSALL HYDROLOGIC SOIL MAP SHEET 12

DRAINAGE AREA = 27.4 Ac

% IMPERVIOUS (POST) = 14%

ROADS = 2460' × 24' = 59,040 S.F

HOUSES & DRIVEWAYS = 21 × 5000# EA = 105,000#

59040 + 105000 = 164,040# = 3.766 Ac

3.766/27.4 = 13.74% SAY 14% ✓

$T_c = T_i + T_t$

T_i = 5 MINUTES FOR INITIAL 1 Ac. LOT

T_t = 1110' ÷ 3'/SEC AVG VELOCITY = 370 SEC ÷ 60 = 6.16 MIN

SAY 6 MIN

T_c = 5 + 6 = 11 MIN

RUNOFF COEF. = .25 FOR PRE & .34 FOR POST CONST

SEE TABLE 3-1

I = 5.5 SEE FIGURE 3-1

DRAINAGE AREA 1A

AREA = 7.9 ACRES

Q PRE = C × I × A = .25 × 5.5 × 7.9 = 10.9 cfs

Q POST = C × I × A = .34 × 5.5 × 7.9 = 14.8 cfs

= 3.9 cfs INCREASE

OFFSITE AREA 1B BY OTHERS

Q = C × I × A = .34 × 5.5 × 4.2 Ac = 7.85 cfs

DRAINAGE AREA 2 - (ENTIRE AREA)

Q PRE = C × I × A = .25 × 5.5 × 19.5 Ac = 26.8 cfs

Q POST = C × I × A = .34 × 5.5 × 19.5 Ac = 36.5 cfs

9.7 cfs INCREASE

DRAINAGE AREA 2A

Q = C × I × A. = .34 × 5.5 × 7.85 = 14.68 cfs

DRAINAGE AREA 2B

Q = .34 × 5.5 × 5.75 = 10.75 cfs

DRAINAGE AREA 2C

Q = .34 × 5.5 × 1.6 Ac = 2.99 cfs

DRAINAGE AREA 2D

Q = .34 × 5.5 × 4.3 Ac = 8.04 cfs

REVIEW OF EXISTING & PROPOSED PIPES

DRAINAGE AREA 1 FLOWS INTO AN EXISTING 30" CMP THAT GOES UNDERNEATH WINTER HAVEN ROAD @ ABOUT A 2% GRADE

TOTAL Q = 1A + OFFSITE = 14.8 cfs + 7.85 cfs = 22.65 cfs

PER THE MANNING EQUATION, A 30" RCP @ 2% HAS A CAPACITY OF 57 cfs

WE HAVE A CMP PIPE ☆ 80% CAPACITY = 45.6 cfs

45.6 cfs > 22.65 cfs = O.K ✓

<u>DRAINAGE AREA 2</u> FLOWS INTO AN EXISTING 24" CMP THAT GOES UNDERNEATH WINTERHAVEN ROAD @ ABOUT A 2% GRADE

TOTAL Q = 36.5 cfs

A 24" CMP @ 2% GRADE HAS A CAPACITY OF ABOUT (.8) 33 cfs = 26.4 cfs < 36.5 cfs

<u>THE PIPE IS UNDERSIZED</u>

A 27" RCP @ 2% HAS A CAPACITY OF 44 cfs

44 cfs > 36.5 cfs = OK ✓

<u>AS A MINIMUM, INSTALL A 27" RCP PIPE</u>

<u>DRAINAGE AREA 2A</u> HAS TO FLOW UNDER THE NEW STREET. THE Q IS 14.68 cfs

A 24" RCP @ 2% HAS A CAPACITY OF 33 cfs

USE A 24" RCP   33 cfs > 14.68 cfs ✓ OK

DRAINAGE AREAS 2A & 2B HAVE TO FLOW UNDER THE NEW STREET. THE Q IS 14.68 + 10.75 = 25.43 cfs

USE A 24" RCP   33 cfs > 25.43 cfs ✓ O.K.

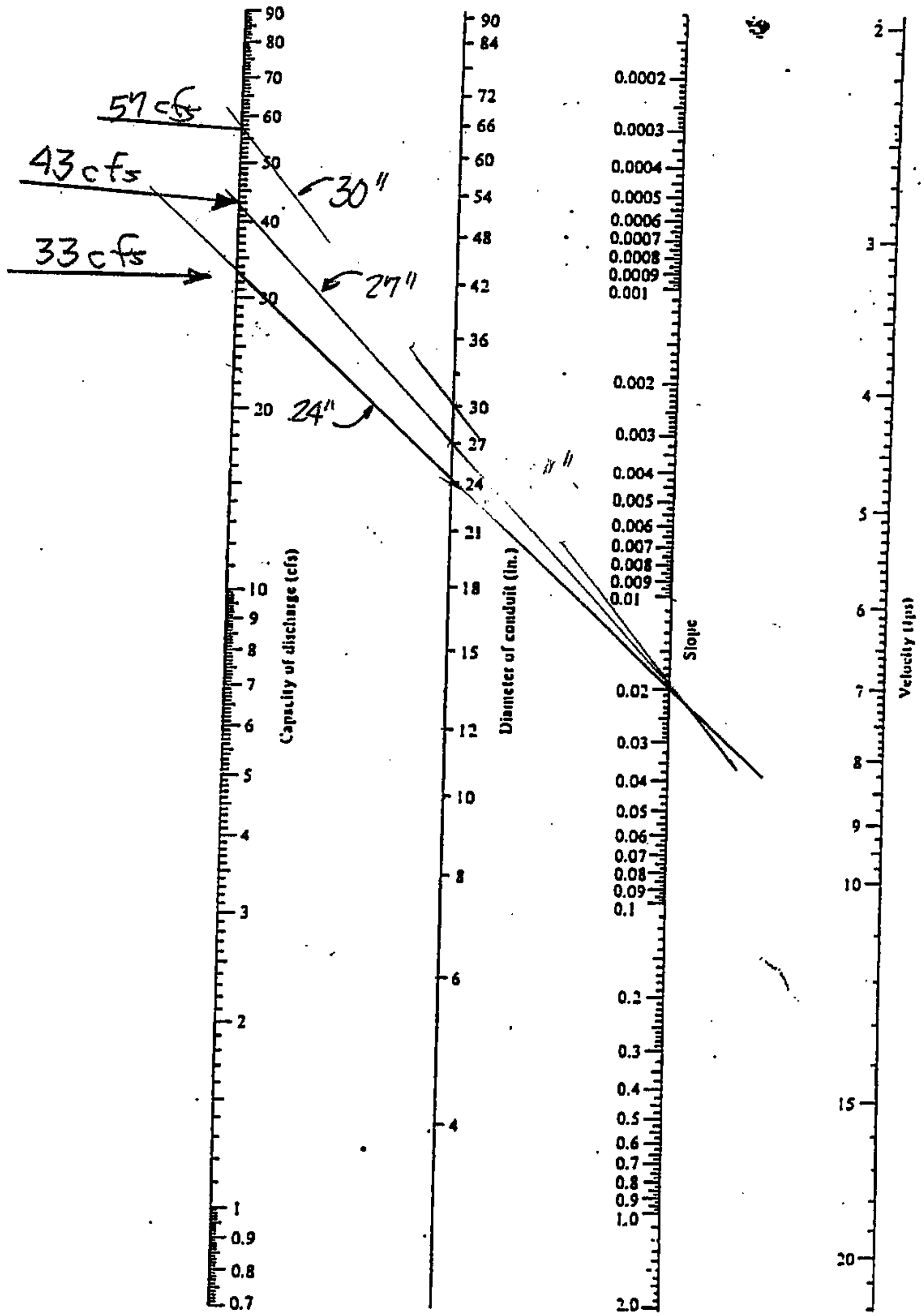
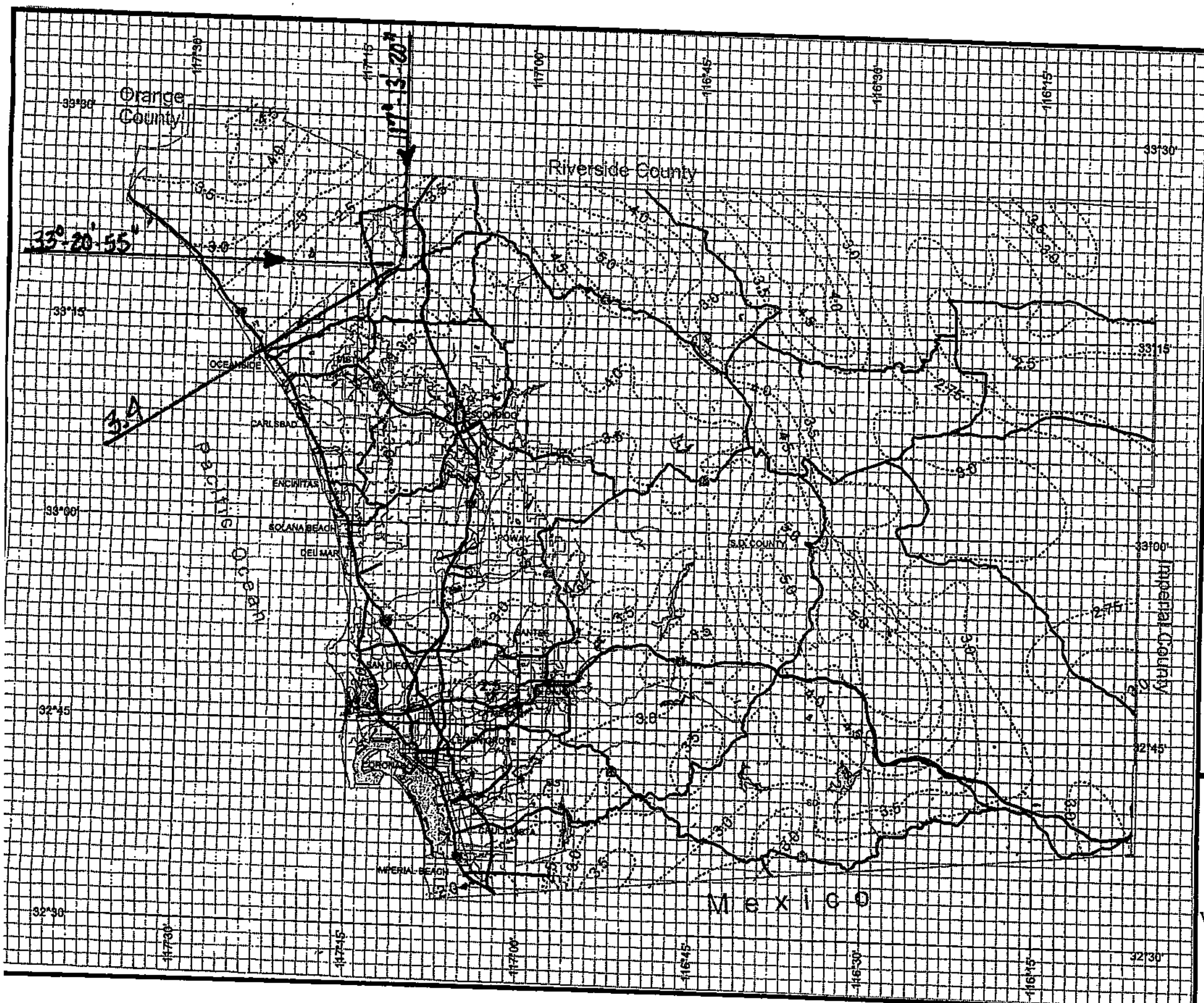


FIGURE 6-1 Nomograph based on Manning's formula for circular pipes flowing full in which $n = 0.013$.



County of San Diego Hydrology Manual

Rainfall Isopluvials

100 Year Rainfall Event - 6 Hours

-------- Isopluvial (inches)

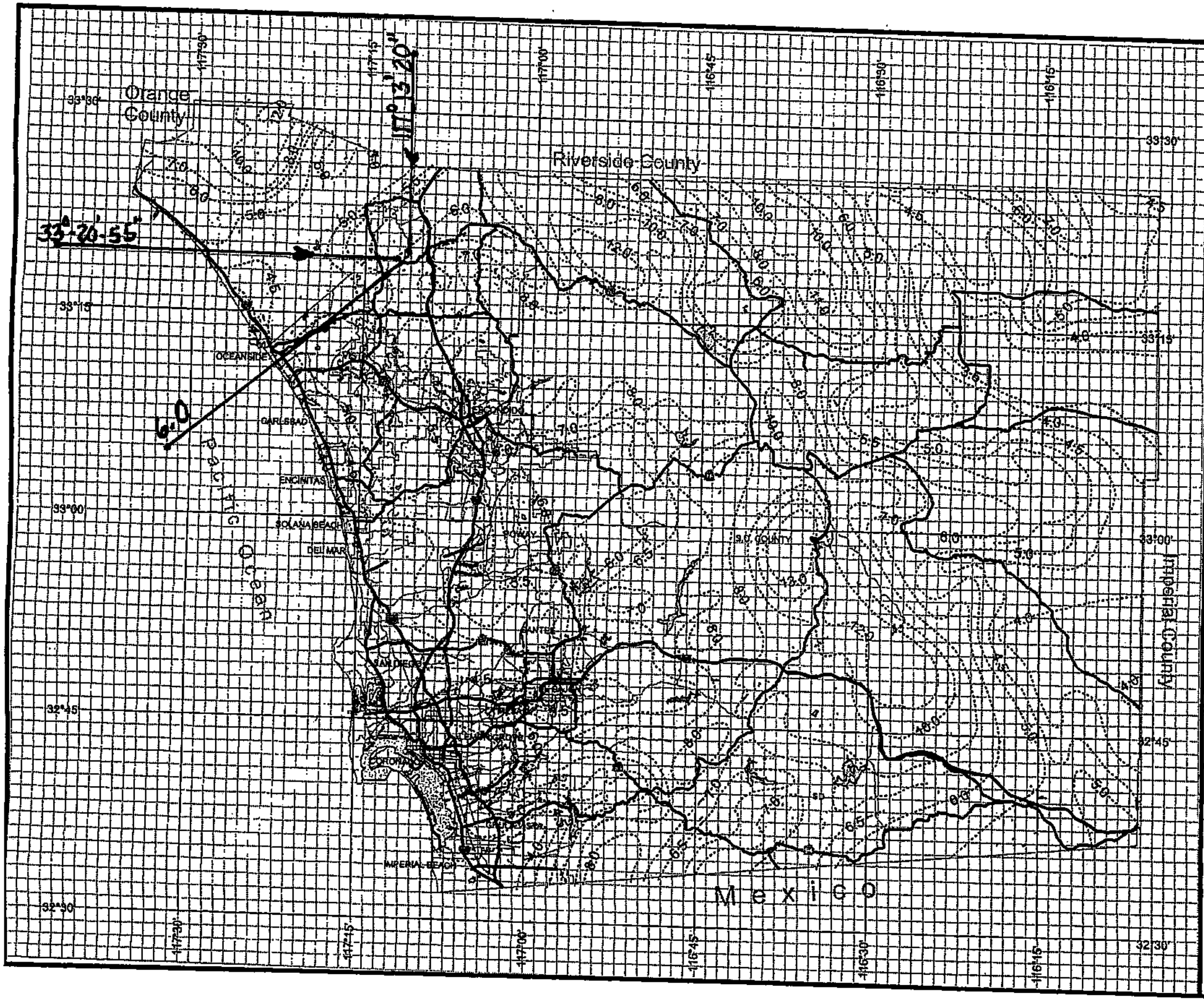
DPW GIS

SanGIS

We Have San Diego Covered!

N S E W

3 0 3 Miles



County of San Diego Hydrology Manual

Rainfall Isopluvials

100 Year Rainfall Event - 24 Hours

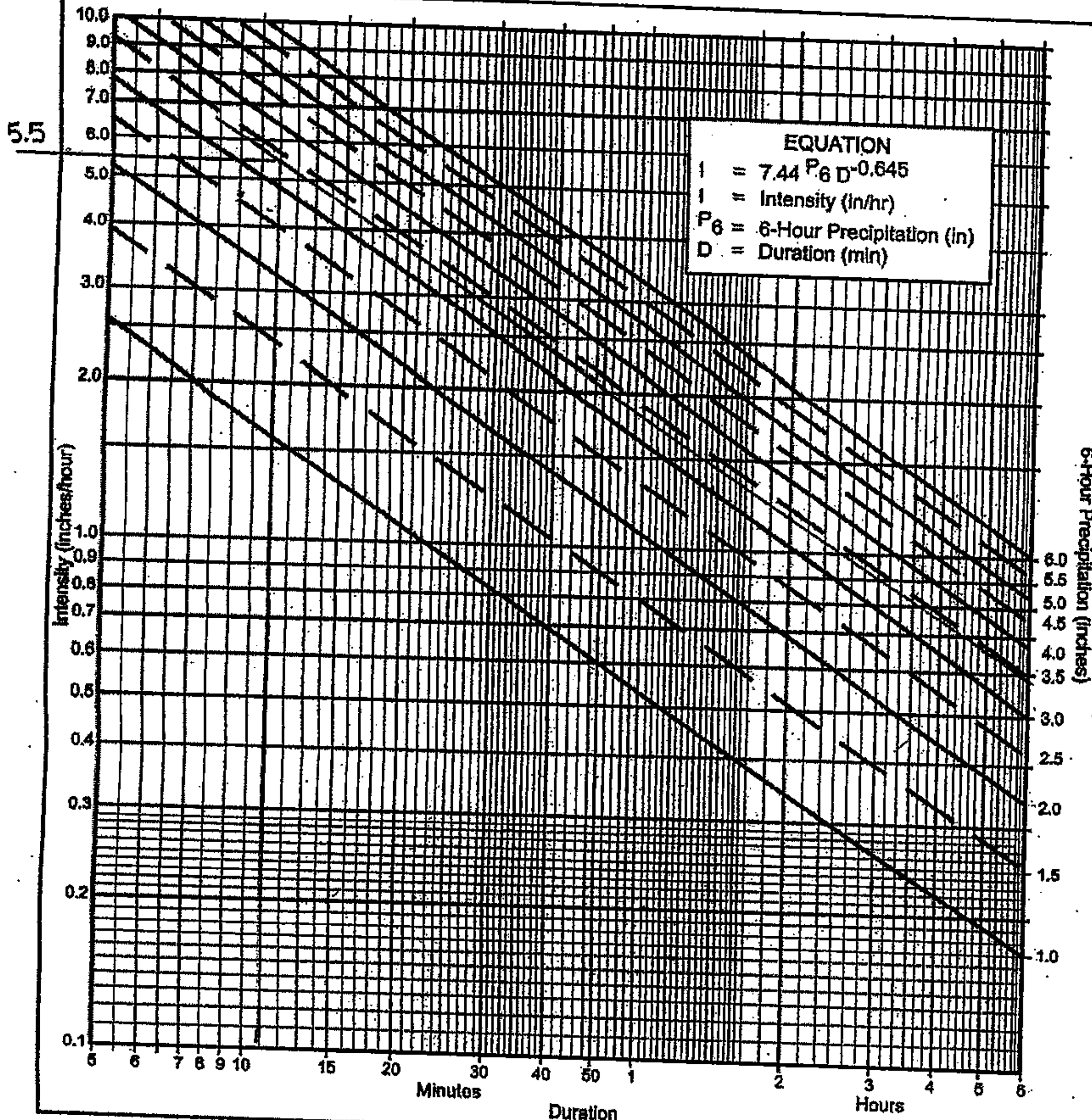
Isopluvial (inches)

DPW GIS

SanGIS We Have San Diego Covered!

N W E S

3 0 3 Miles



Directions for Application:

(1) From precipitation maps determine 6 hr and 24 hr amounts for the selected frequency. These maps are included in the County Hydrology Manual (10, 50, and 100 yr maps included in the Design and Procedure Manual).

(2) Adjust 6 hr precipitation (if necessary) so that it is within the range of 45% to 65% of the 24 hr precipitation (not applicaple to Desert).

(3) Plot 6 hr precipitation on the right side of the chart.

(4) Draw a line through the point parallel to the plotted lines.

(5) This line is the intensity-duration curve for the location being analyzed.

Application Form:

(a) Selected frequency 100 year

(b) P_6 = 3.4 in., P_{24} = 6.0, $\frac{P_6}{P_{24}}$ = 57 %(2)

(c) Adjusted $P_6^{(2)}$ = 3.4 in.

(d) t_x = 11 min.

(e) I = 5.5 in./hr.

Note: This chart replaces the Intensity-Duration-Frequency curves used since 1965.

P6	1	1.5	2	2.5	3	3.5	4	4.5	5	5.5	6
Duration	I	I	I	I	I	I	I	I	I	I	I
5	2.63	3.95	5.27	6.59	7.90	9.22	10.54	11.86	13.17	14.49	15.81
7	2.12	3.18	4.24	5.30	6.36	7.42	8.48	9.54	10.60	11.66	12.72
10	1.68	2.53	3.37	4.21	5.05	5.90	6.74	7.58	8.42	9.27	10.11
15	1.30	1.95	2.59	3.24	3.89	4.54	5.19	5.84	6.49	7.13	7.78
20	1.08	1.62	2.15	2.69	3.23	3.77	4.31	4.85	5.39	5.93	6.46
25	0.93	1.40	1.87	2.33	2.80	3.27	3.73	4.20	4.67	5.13	5.60
30	0.83	1.24	1.66	2.07	2.49	2.90	3.32	3.73	4.15	4.56	4.98
40	0.69	1.03	1.38	1.72	2.07	2.41	2.76	3.10	3.45	3.79	4.13
50	0.60	0.90	1.19	1.49	1.79	2.09	2.39	2.69	2.98	3.28	3.58
60	0.53	0.80	1.06	1.33	1.59	1.86	2.12	2.39	2.65	2.92	3.18
90	0.41	0.61	0.82	1.02	1.23	1.43	1.63	1.84	2.04	2.25	2.45
120	0.34	0.51	0.68	0.85	1.02	1.19	1.36	1.53	1.70	1.87	2.04
150	0.29	0.44	0.59	0.73	0.88	1.03	1.18	1.32	1.47	1.62	1.76
180	0.26	0.39	0.52	0.65	0.78	0.91	1.04	1.18	1.31	1.44	1.57
240	0.22	0.33	0.43	0.54	0.65	0.76	0.87	0.98	1.08	1.19	1.30
300	0.19	0.28	0.38	0.47	0.56	0.66	0.75	0.85	0.94	1.03	1.13
360	0.17	0.25	0.33	0.42	0.50	0.58	0.67	0.75	0.84	0.92	1.00

Intensity-Duration Design Chart - Template

FIGURE

3-1

HYDROMODIFICATION ANALYSIS FOR BERK SUBDIVISION COUNTY OF SAN DIEGO, CA

INTRODUCTION

This memorandum summarizes the approach used to model the proposed housing development project site in Fallbrook, under the jurisdiction of San Diego County, using the Environmental Protection Agency (EPA) Storm Water Management Model 5.0 (SWMM). SWMM models were prepared for the pre and post-developed conditions at the site in order to determine if the proposed LID bioretention facilities have sufficient volume to meet the current Hydromodification Management Plan (HMP) requirements from the San Diego Regional Water Quality Control Board (SDRWQCB).

SWMM MODEL DEVELOPMENT

The Berk Subdivision project proposes the creation of a residential development with 21 houses in the NW corner of Winter Haven Rd and Sunnycrest Ln in Fallbrook. Two (2) SWMM models were prepared for this study: the first for the pre-development and the second for the post-developed conditions. The project site drains to two (2) Points of Compliance (POC-1 and POC-2) located at the existing low points in Winter Haven Rd, to the south of the project site.

The SWMM model was used since we have found it to be more comparable to San Diego area watersheds than the alternative San Diego Hydrology Model (SDHM) and also because it is a non-proprietary model approved by the HMP document. For both SWMM models, flow duration curves were prepared to determine if the proposed HMP facility is sufficient to meet the current HMP requirements.

The inputs required to develop SWMM models include rainfall, watershed characteristics, and BMP configurations. The Fallbrook Gage from the Project Clean Water website was used for this study, since it is the most representative of the project site precipitation due to elevation and proximity to the project site.

Evaporation for the site was modeled using average monthly values from the County hourly dataset. The site was modeled with Type D and Type C hydrologic soils as these are the existing soils determined from the NRCS Soil Survey. Soils have been assumed to be un-compacted in the existing condition to represent the current condition of the site and fully compacted in the post developed conditions. Other SWMM inputs for the subareas are discussed in the appendices to this document, where the selection of the parameters is explained in detail.

HMP MODELING

UNDEVELOPED CONDITIONS

Currently, in the existing site, there is a very small portion of the area that corresponds to impervious surfaces along the boundary roads; however, as a conservative assumption, those impervious areas are not included in the model (pre-development conditions are assumed 100% pervious).

DEVELOPED CONDITIONS

Storm water runoff from the proposed project site is routed to two (2) POC located at the low points of the existing Winter Haven Rd to the south of the project site. Runoff from the developed project site is drained to two (2) onsite receiving bioretention LID BMPs, one at each POC. Once flows are routed via the proposed LID BMPs, developed onsite flows are then conveyed to the culvert crossings within the adjacent Winter Haven Road. Additional runoff from adjacent areas to the project site are unable to be routed to the BMPs (areas 1-3C, 1-4D and 1-5C for POC-1 and areas 2-3C, 2-4D and 2-5C for POC-2 are by-passed). The impervious areas of those by-passed sub-areas, is equal to the impervious area in pre-development conditions (as the roads do not change); therefore, they are also not included in the modelling effort, similarly as assumed for pre-development conditions. Impervious roads could have been removed from the analysis in both pre and post-development conditions, but for simplicity in terms of the areas depicted, the impervious areas were included as pervious in the total contributing area.

Two (2) LID bioretention basins are located within the project site and are responsible for handling hydromodification requirements for the project site. In developed conditions, the basins will have a surface depth of 1.0 ft for BMP-1 and 2.5 feet for BMP-2 and a variable surface outlet structure (see dimensions in Table 1). Flows will then discharge from the basin via a low flow orifice(s) outlet within the gravel layer. The riser structure will act as a spillway such that peak flows can be safely discharged to the receiving storm drain system.

Beneath the basins' invert lies the proposed LID bioretention portion of the drainage facility. This portion of the basin is comprised of a 2-inch layer of mulch, an 18-inch layer of amended soil (a highly sandy, organic rich composite with an infiltration capacity of at least 5 inches/hr) and an 12-inch layer of gravel (BMP-1) or a 15-inch layer of gravel (BMP-2) for additional detention and to accommodate the French drain system. These systems are to be located beneath the bioretention layers to intercept treated storm water and convey these flows to a small diameter lower outlet orifice(s): the lower orifice will be 1 orifice, 2 inches in diameter for BMP-1, or two orifices, 3.75 inches in diameter each for BMP-2. Once flows have been routed by the outlet structure, flows are then discharged to the existing storm drain located within Ocean Ranch Boulevard.

The bioretention basins were modeled using the bioretention LID module within SWMM. The bioretention module can model the underground gravel storage layer, underdrain with an orifice plate, amended soil layer, and a surface storage pond up to the elevation of the invert of the spillway.

It should be noted that detailed outlet structure location and elevations will be shown on the construction plans based on the recommendations of this study.

BMP MODELING FOR HMP PURPOSES

Modeling of dual purpose Water Quality/HMP BMPs

Two (2) LID BMP bioretention basins are proposed for water quality treatment and hydromodification conformance for the project site. Table 1 illustrates the dimensions required for HMP compliance according to the SWMM model that was undertaken for the project.

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TABLE 2 – SUMMARY OF DEVELOPED DUAL PURPOSE BMPs

BMP	DIMENSIONS							
	BMP Area[1], (ft^2)	Surface[1] elev. with $A = A_{BMP}$	Gravel Depth[2] (in)	Lower Orif. D (in)[3]	Middle Orif. (number & D, inches)[4]	Riser Invert (in)[5]	Weir Length[6] (ft)	Total Surface Depth[7] (in)
1	2060	3 inches	12	1 orifice, 2 inches	N/A	6	12	1.0
2	8000	9 inches	15	2 orifices, 3.75" each	10, 4" each	19	12	2.5

Notes:
(1): Area of amended soil equal to area of gravel = area at the surface elevation indicated
(2): Gravel depth needed to comply with hydromodification purposes
(3): Diam. of orifice(s) in gravel layer with invert at bottom of layer; tied with hydromod min threshold ($0.1 \cdot Q_2$).
(4): Orifices at middle level, only for BMP-2 (see detail)
(5): Depth of ponding beneath riser structure's surface spillway.
(6): Overflow length, the internal perimeter of the riser is 12 ft (3 ft x 3 ft internal dimensions).
(7): Total surface depth of BMP from top crest elevation to surface invert.

FLOW DURATION CURVE COMPARISON

The Flow Duration Curve (FDC) for the site was compared at the POC-1 and POC-2 by exporting the hourly runoff time series results from SWMM to a spreadsheet. The FDC was compared between 10% of the existing condition Q_2 up to the existing condition Q_{10} for both POC-1 and POC-2. The Q_2 and Q_{10} were determined with a partial duration statistical analysis of the runoff time series in an Excel spreadsheet using the Cunnane plotting position method (which is the preferred plotting methodology in the HMP Permit). As the SWMM Model includes a statistical analysis based on the Weibull Plotting Position Method, the Weibull Method was also used within the spreadsheet to ensure that the results were similar to those obtained by the SWMM Model.

The range between 10% of Q_2 and Q_{10} was divided into 100 equal time intervals; the number of hours that each flow rate was exceeded was counted from the hourly series. Additionally, the intermediate peaks with a return period "i" were obtained (Q_i with i=3 to 9). For the purpose of the plot, the values were presented as percentage of time exceeded for each flow rate. FDC comparison at each POC is illustrated in Figures 1 and 2 in both normal and logarithmic scale. Attachment 5 provides a detailed drainage exhibit for the post-developed condition.

As can be seen in Figures 1 and 2, the FDCs for the proposed condition with the HMP BMPs are within 110% of the corresponding curve for the existing condition in both peak flows and durations. The additional runoff volume generated from developing the site will be released to the existing point of discharge at a flow rate below the 10% Q_2 lower threshold for both POCs. Additionally, the project will also not increase peak flow rates between the Q_2 and the Q_{10}, as shown in the graphic and also in the peak flow tables in Attachment 1.

SUMMARY

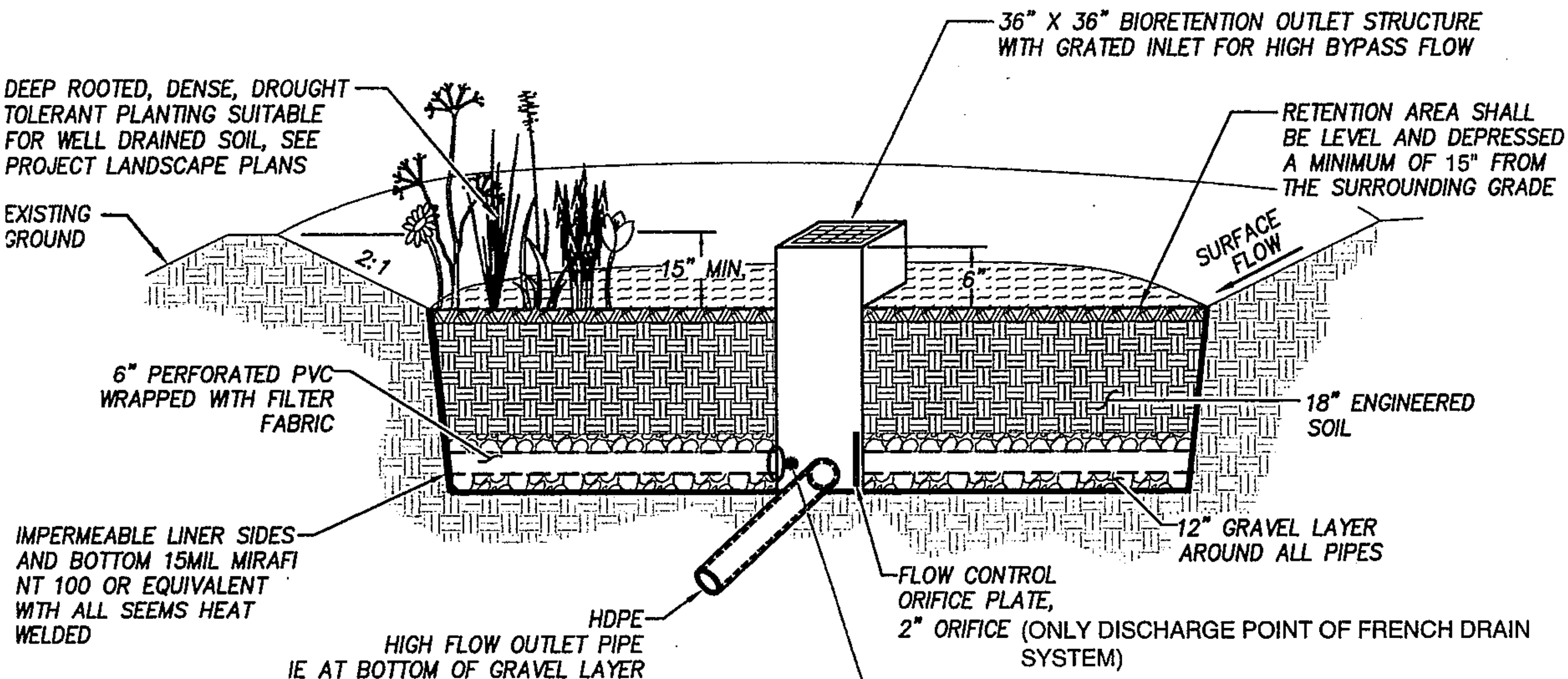
This study has demonstrated that the proposed HMP BMPs provided for the Berk subdivision site is sufficient to meet the current HMP criteria if the cross-section areas and volumes recommended within this technical memorandum, and the respective orifices and outlet structure are incorporated as specified within the proposed project site.

KEY ASSUMPTIONS

1. Type C and D Soil are representative of the existing condition site.
2. SWMM Model used to insure Hydromodification Compliance of areas draining to the POCs studied. Areas not draining to BMP (Lot 19) to be satisfied by designing a BMP facility in this lot in accordance to the HMP Tables of the HMP Document, San Diego County.

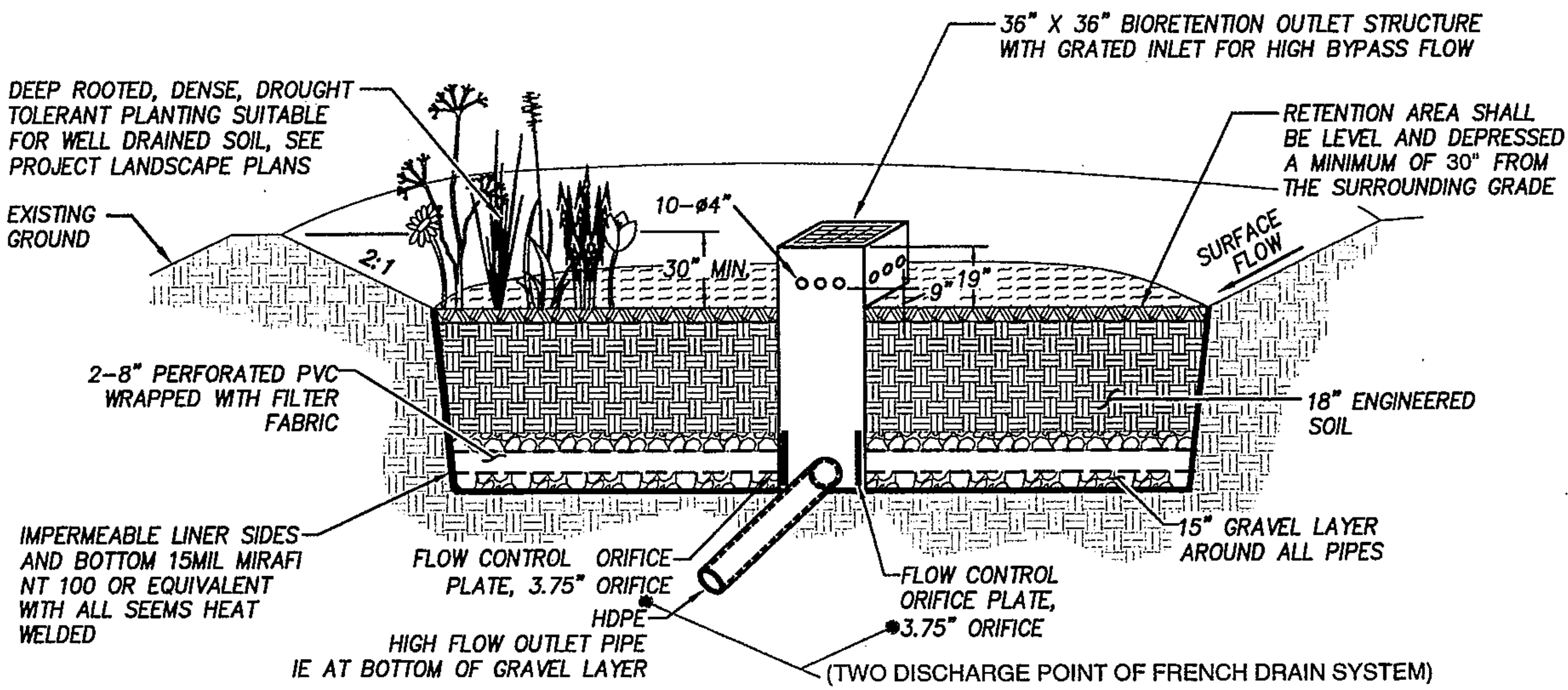
ATTACHMENTS

1. Q_2 to Q_{10} Comparison Tables
2. FDC Plots (log and natural "x" scale) and Flow Duration Table.
3. List of the "n" largest Peaks: Pre-Development and Post-Development Conditions
4. Elevations vs. Discharge & Stage- Storage Curves to be used in SWMM
5. Pre & Post Development Maps, Project Plan and section sketches
6. SWMM Input Data in Input Format (Existing and Proposed Models)
7. SWMM Screens and Explanation of Significant Variables
8. Soil Maps
9. Summary files from the SWMM Model
10. Response to Comments



BMP 1
PRIVATE BIORETENTION FACILITY DETAIL

NOT TO SCALE



BMP 2
PRIVATE BIORETENTION FACILITY DETAIL

NOT TO SCALE

6/6

EXHIBITS